Chapter 13
A Case Study for Seismic Assessment and Restoration of Historic Buildings:
The Arditi Residence

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ABSTRACT

This chapter focuses on seismic assessment and restoration of one of the numerous historical buildings in Turkey; the Arditi Residence in Urla, Izmir. It is a 150 years old two story masonry building located in a seismically active region. From the structural point of view, the building can be regarded as a mixed system since three different techniques had been used during the construction. The Arditi Residence has been investigated in three stages: preliminary evaluation, seismic performance assessment and intervention. The building has been observed to possess serious deficiencies, which are not easy to handle due to the complexity of the construction system. On the other hand, the proposed intervention strategies should have the minimal impact on the historic information building carrying and provide a certain level of safety against the seismic demands. Overall, the chapter presents a contribution to seismic assessment and restoration of historical structures on the basis of Arditi Residence, a unique historical building with serious problems in an earthquake-prone region.

INTRODUCTION

It is always an important task to assess the present condition of existing historic structures and to develop intervention strategies if required for preservation and sustainability of cultural heritage. This becomes even more crucial in earthquake prone regions. The most challenging part of the problem is that every historic structure is unique and possesses its own structural characteristics. Hence it is not

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possible to develop and employ a standard procedure for the seismic assessment and rehabilitation of historic structures. There are many studies in the literature regarding the condition and seismic safety assessment of historical masonry structures (Asteris *et al.* 2005, Asteris 2008, Betti *et al.* 2010, Betti *et al*. 2011, Asteris *et al.* 2014).

There are numerous historic structures and monuments in Turkey in accordance with the cultural wealth and diversity over the centuries. On the other hand, Turkey is a seismically active country, where a major earthquake occurs in every decade. Therefore one of the privileged missions in earthquake mitigation studies in Turkey is to assess the seismic performance of historic structures and develop rehabilitation techniques for their preservation. This study focuses on one of these historical buildings, the Arditi Residence in Urla, Izmir.

The Arditi Residence is a two story building with a partial basement. The building, which is nearly 150 years old, had been constructed for one of the most prominent families of that region in that era. It was used as a residence since 1930s, after which it was used by the local authorities as a public building. From the structural point of view, the building can be regarded as a mixed system since three different techniques had been used during the construction.

The building is located in a seismically active region. In the last decade, the building experienced four earthquakes with moment magnitudes ranging between 5.0 and 6.0. The preliminary assessment of the building reveals that it had been slightly affected by these small-to-moderate magnitude events. Due to the seismotectonic characteristics of the region, it is probable that the building may experience an earthquake with the order of magnitude 6.5-7.0 in the future.

The Arditi Residence has been investigated in three stages: preliminary evaluation, seismic performance assessment and intervention. The preliminary evaluation stage started with a building site visit. The conditions of all structural components were assessed and the damaged components were recorded. The observations indicated that although the building had not been subjected to any severe seismic action, it possessed serious problems due to poor maintenance over the years. Some of the walls were not able to maintain their structural load-carrying capacity due to degraded material properties. Existing crack patterns reveals that wall-to-wall and wall-to-floor connections of the building were poor. The cross sectional sizes and the connection details of the timber floor indicates that it was not able to exhibit the rigid floor diaphragm action. This deficiency endangers the even distribution of shear forces in accordance with the relative rigidities of structural components.

In order to quantify these observations in a detailed manner, seismic performance assessment of the building was carried out in the second stage. To achieve this, the current Turkish seismic code was considered as a reference. At the end of the assessment, it was observed that existing structural walls in the building was insufficient for the seismic hazard level of a design earthquake (with a return period of 475 years). Details are provided in the assessment of the existing condition of the building section.

After preliminary and detailed evaluation, the structural deficiencies of the building were assessed and some unique intervention strategies were developed. Depending on the impact of the deficiency, intervention strategies are grouped into two sets. Deficiencies that are creating local effects such as if it is related to a member or a limited portion of the structure, it is investigated under the member level interventions. On the other hand, if the deficiency is causing a global effect on the structure and its solution needs multi member interventions, it is called structure level interventions. The details of the interventions are explained in detail in last section of this chapter.

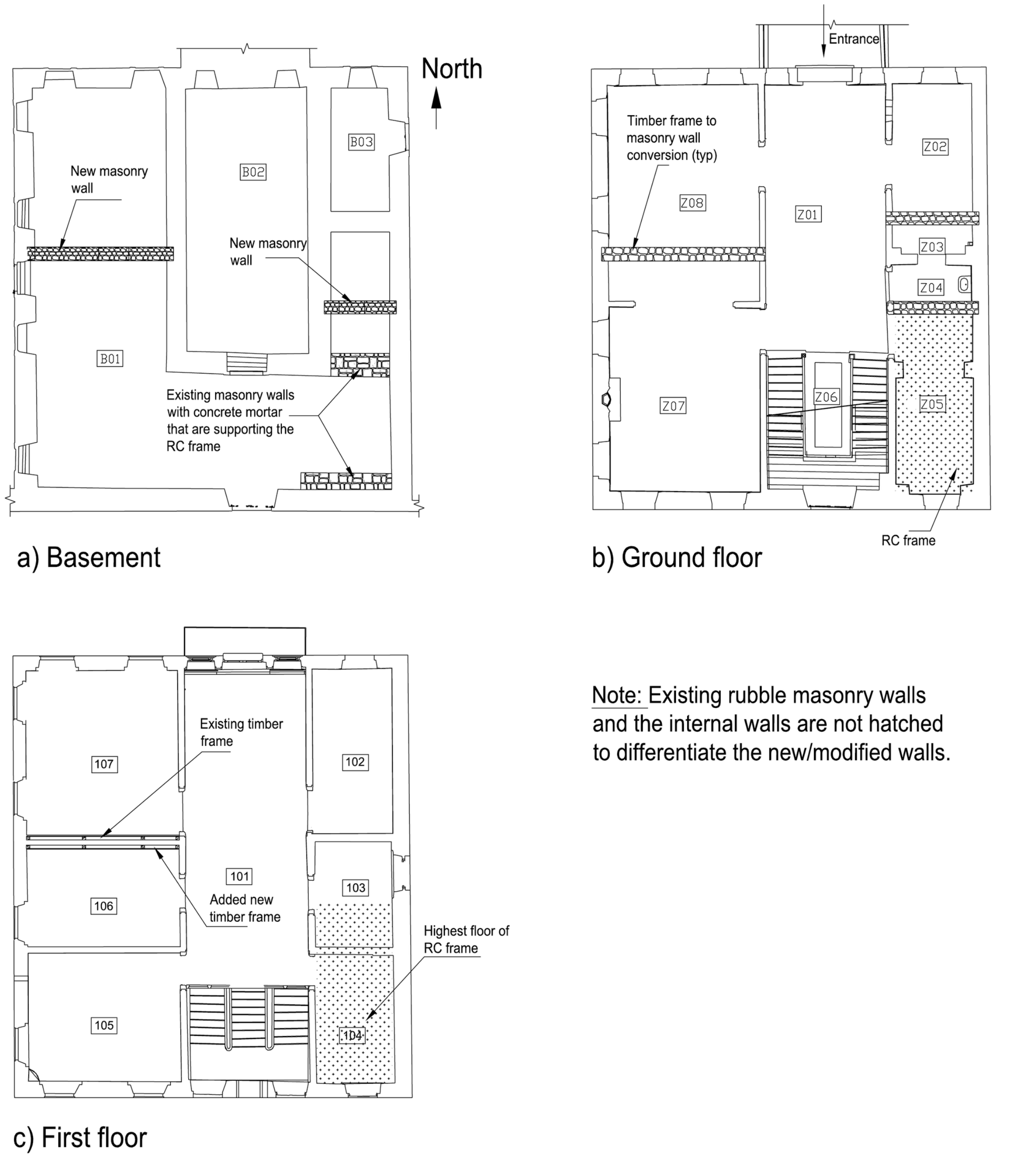
FIELD INVESTIGATIONS

In the first stage of the investigation, preliminary assessment of the building was carried out by field investigations. As seen in Fig. 1, the Arditi Residence is a two story building with a partial basement. The building, which is nearly 150 years old, has an almost rectangular structural plan. Story level plans of the building are presented in Fig. 2. Although it seems like a masonry building, structure should be regarded as a mixed system since three different techniques had been used during the construction. The basement walls had been constructed by rubble stone whereas the periphery walls in the ground and the first floor had been constructed as timber framed stone masonry walls. This system is called "hımış" locally. Considering some of the interior walls in the ground floor and all of the interior walls in the first floor, a local and traditional construction technique, timber frame with "bağdadi" covering, is used. This is similar to timber lath and plaster technique. In both stories, the structural walls had been connected to a timber floor system. The building has a hipped roof covered by clay tiles. The existing conditions of structural members in each story are explained in detail in the following paragraphs.

In the basement, about 60 cm thick peripheral walls are made of rubble stone. There exist two rows of wooden horizontal lintel bands, wall plate timbers, with a vertical spacing of approximately 2 meters. At the observable locations, it is seen that the bottom layer of wall plate timbers are totally degraded and almost empty shafts existed in these locations, Fig. 3a. Such a heavy degradation is due to the proxim-

Figure 1. A photo of the Arditi Residence from south-east direction

Figure 2. Floor plans of the building



ity of these timber members to the moist soil. Although some of the wall plate timbers had been worn out due to weathering effects for a long period of time, existing timber still has a significant effect on the integrity of the basement walls. Other than the peripherals walls, there are two parallel walls in the north-south (N-S) direction of the building, see Fig. 2 basement plan. These walls are in the same vertical alignment with the ground floor interior walls that enclose the entrance hallway of the building. There exist no interior walls in the east-west (E-W) direction in the basement floor.

Figure 3. Typical damages observed in the case study building; a) deteriorated masonry wall ties in basement floor, b) deteriorated timber due to insect activity, c, d) deteriorated wall section at the north-east corner due to the crack that permits moisture to penetrate, e, f) typical cracks at the top of the windows

a) Deteriorated masonry wall ties, basement floor

b) Deteriorated timber due to insect activity

c and d) Deteriorated wall section at the north-east corner due to the crack that permits moisture to penetrate

e and f) Typical cracks at the top of the windows

During the field investigation, it was observed that lime mortar had been used in the construction of the basement walls. In some parts of the basement walls, there are fine cracks in the vertical direction, which seem to be caused by some differential movement at the base of the building possibly due to the ground settlement.

In the ground floor, which has a story height of 4.5 meters, there exist three different types of walls. The perimeter walls had been constructed as two leaves. The exterior leaf is made of rubble stone with lime mortar whereas the inner leaf has a timber frame infilled by rubble stone. The total wall thickness is about 50 cm. The inner walls, with a thickness of approximately 20 cm, had been constructed either as timber framed walls infilled with rubble stone or timber frame with "bağdadi" covering, which is a traditional construction technique from Ottoman era. This is similar to timber lath and plaster technique. It has been observed that the buildings constructed by using this technique behaved in a satisfactory manner during major earthquakes (Gulkan & Langenbach 2004, Guchan 2007). All the structural walls had been connected to a timber floor system. By removing the plaster in few regions of the walls, it was observed that some timber members had been worn out due to weathering, insect and fungal activity, Fig. 3b. In the regions where the cracks at the perimeter walls permit water penetration, heavy deteriorations are observed, Fig. 3 c and d. In some of the walls, shear cracks were encountered. There were extensive cracks in the connections of perpendicular walls, especially between the peripheral and interior walls. This reveals that wall-to-wall connections are poor. Furthermore, the cracks at the bottom parts of the structural walls are indications of improper wall-to-floor connection. There are also cracks extending from the corners of window and door openings toward the spandrels in the peripheral walls, Fig 3e and f.

In the first floor, the thickness of the peripheral wall is reduced to 45 cm. The story height is 3.80 meters. There are two different types of structural walls: perimeter walls with two leaf rubble stone and inner walls with" bağdadi" covering. Similar to the ground floor, cracks at wall-to-wall and wall-to-floor connections were encountered in this story. Roof slab was also partially damaged and the roof material was deteriorated due to rain water penetration.

At the south-east part of the ground and first story floors, there exists a partial reinforced concrete frame, which seems to be added to the original framing after initial construction date. This partial construction contradicts with the remaining part of the building from structural point of view. It impairs the structural performance of the building since it affects the force distribution and stress transfer in an adverse way due to its completely different stiffness and strength characteristics.

ASSESSMENT OF THE EXISTING CONDITION OF THE BUILDING

In the second stage of the investigation, seismic performance of the Arditi residence is assessed in accordance with the recommendations of the current Turkish earthquake code (2007). Surely, it is not expected for an old masonry building to comply with the rules of the current seismic code completely, but such an assessment could give an indication about the current condition of the considered building in terms of its seismic resistance, and whether it conforms to the basic rules of earthquake resistant construction or not.

Western Anatolia is a seismically active region of Turkey with different fault systems. In Urla region, where the case study building is located, it is known that many strong earthquakes had occurred. Considering the historical earthquakes that affected the region, the most significant one had occurred in 1778 (Ambraseys & Finkel, 1995). It is rumored that the coastline of Urla peninsula had been flooded after the earthquake. In the near future, a moderate earthquake occurred in the region in 2003 with moment magnitude M_w=5.7. Then, in 2005, a series of earthquakes took place within four days with moment magnitudes of 5.0, 5.8, 5.5 and 5.9, respectively. The observations revealed that the 2003 earthquake only caused slight damage in the buildings of the region (Emre *et al*. 2005). During the sequential earthquakes in 2005, the number of severely, moderately and slightly damage buildings in Urla center was reported as 72, 90 and 192, respectively (Denizlioglu *et al.* 2006). There is no information related to the performance of the case study building during these earthquakes. On the other hand, observations in the 1950's era neighboring reinforced concrete building that was inhabited in the last decade show some evidence of seismic damage. This is a three story moment frame tobacco storage structure. The solid brick infills walls at the ground floor that are separating the storage area from the stairs have about 1 to 5 mm thick web of diagonal cracks which can typically be caused by seismic loading. The field observations of the studied building indicate that it did not experience any significant damage.

From the structural point of view, peripheral two leaf stone masonry walls can be regarded as the main load-carrying system for the building. The out of plane action of the peripheral walls needed to be prevented by the connecting inner walls and the floor diaphragm. The inner walls in the N-S and E-W directions are not continuous in the elevation, so these walls cannot be assumed as load-carrying members. Hence, it is assumed that the thick peripheral walls try to resist the lateral earthquake forces and keep the integrity of the building with box action. Here, box action means the resistance of in-plane and out-of-plane loaded masonry walls as a single system against lateral forces. However it should be pointed out that in order to ensure such a box effect, wall-to-wall and wall-to-floor connections of the building should be well detailed and in a good state. Due to the extensive cracking observed in these regions, the connections are not good enough to exhibit such a desirable behavior in their current states.

The case study building, which is located in the most severe seismic zone according to the current Turkish earthquake code, is a two-story structure with a partial basement. From this point of view, it can be stated that the building conforms to the seismic code since the maximum number of stories allowed for masonry buildings in the most severe seismic zone is two. Here, it is assumed that the partial basement has no contribution to the vibration characteristics of the building.

The length of N-S and E-W (parallel to the road) façades of the building are approximately 13 and 12 meters, respectively. Hence the plan area of the building is calculated as 156 m^2. As a conservative assumption, the weight of a typical floor (including the live load) is taken as 10 kN/m^2 whereas it is taken as 5 kN/m^2 for the roof. By using these values, the total weight of the building is obtained as 3,900 kN. A more detailed study, through calculating the weights of the individual walls and floor framing by using the material and thickness information resulted to a total weight of the building as 3,650 kN.

There exist some basic geometrical limitations for structural masonry walls in the current seismic code. Among these, the most important criterion is the one related to the total length of the load bearing walls in any principal direction. Accordingly, the ratio of the total length of masonry load-bearing walls in any of the orthogonal directions in plan (excluding window and door openings) to gross floor area (excluding cantilever floors) should be equal to or greater than 0.2I (in m/m^2), where I represents building importance factor in the seismic code and it is equal to unity for residential buildings. When this criterion is checked for the case study building, it is observed that the ratios in N-S and E-W directions are obtained as 0.14 m/m^2 and 0.07 m/m^2 respectively. Both values are smaller than the limiting value of 0.2 m/m^2. Although the results indicate that the building does not have adequate length of load-bearing masonry walls in both principal directions this may be misleading since the walls are very thick and this criterion in the code does not take the thickness of the wall into account. A more objective comparison can be made by considering a similar criterion in Eurocode 8 (CEN 2003), which had been proposed for "simple masonry buildings". This criterion considers not the total length but the total cross-sectional area of the load bearing walls in any of the orthogonal directions in plan to the gross floor area. According to Eurocode 8, in seismic regions where the effective ground acceleration exceeds 0.3g, the ratio is expected to be larger than 6%. When this new parameter is used for the case study building, the ratios for N-S and E-W directions are calculated as 6.3% and 2.9%, respectively. Same ratios are about 5.7% and 2.6% for first story of the building. This indicates that, considering also the thickness of the peripheral walls and decrease of the lateral forces at the first floor, the amount of structural walls in the N-S direction seems to be adequate whereas the walls in the E-W direction are not enough to ensure the lateral load carrying capacity of the building.

Other geometrical limitations regarding structural masonry walls in the current version of the seismic code includes the locations of the openings in the walls. Accordingly, the code enforces minimum length requirements for wall segments between any two openings or between an opening and corner of the building. These requirements are violated in some parts of the peripheral walls but these violations do not seem to be very critical when the thickness of the peripheral walls is considered. There is another criterion regarding the unsupported length of structural walls between the connecting wall axes in the perpendicular direction, which shall not exceed 5.5 m in the most severe seismic zone. This criterion is theoretically not violated in the case study building. However, from practical point of view, since wall-to-wall connections are poor in the building, there may still be the potential of out-of-plane damage during a strong earthquake.

There exist different computational strategies to assess seismic performance of masonry buildings in the literature. It is possible to model these structures as equivalent frame systems whereas one can also use detailed finite element models (Kappos *et al*. 2002). There is also a wide range of analysis techniques from linear static analysis to nonlinear dynamic analysis to be used for this purpose. Seismic codes and standards generally recommend the most basic ones among these approaches and techniques. Accordingly, the Turkish earthquake code includes rudimentary stress calculations to check whether the building is safe under vertical and horizontal loading. These stress calculations are generally carried out for the most critical story of the building. In the case study building, the most critical story is considered as the ground story. During calculations, only the contribution of peripheral masonry walls to the vertical and lateral load carrying capacities are considered. The inner walls are not taken into account in a conservative manner. Hence the vertical stress on peripheral walls (σ) can easily be calculated by dividing the weight to the total cross-sectional wall area (excluding the openings), yielding a value of 0.28 MPa. This value is then compared with the allowable compressive stress (f_{em}) which is referred in the code as a function of masonry unit and mortar characteristics.

$$R \cdot f_{em} > \sigma \tag{1}$$

In Equation 1, *R* is a factor in order to reduce the allowable compressive stress as a function of the slenderness ratio (i.e. the ratio of wall height of thickness) of the wall. Considering all these parameters, the reduced allowable stress (i.e. the left hand side of Equation 1) is obtained as 0.27 MPa for the case study building. Hence the criterion in Equation 1 seems to be not satisfied although the values are very close to each other. However it should be kept in mind that if the contribution of inner walls is included, the allowable stress would exceed the vertical stress.

For the shear stress calculations, first the base shear force (V_b) should be calculated by using the formulation given in Equation 2 as proposed by the code.

$$V_b = \frac{W A_0 I S(T_1)}{R_a(T_1)} \tag{2}$$

In this equation, W is the building weight, A_0 is the effective ground acceleration coefficient, I is the building importance factor, $S(T_1)$ is the spectrum coefficient at the fundamental period of the building and $R_a(T_1)$ is the seismic load reduction factor. As recommended by the code for masonry buildings, parameters $S(T_1)$ and $R_a(T_1)$ are taken as 2.5 and 2.0, respectively. Hence the base shear force is calculated as 1625 kN for the building. The next step is to distribute this shear force to the structural walls in accordance with their relative rigidities, by also considering the additional shear forces due to torsional irregularities of the building under consideration. This analysis is based on the assumption that the floors are rigid enough to enable the distribution of shear forces to the structural walls. In code-based approaches, such a simplified assumption can be justified. However if more detailed analysis are required in the case of historical masonry buildings with flexible floor diaphragms, different approaches should be used (Betti *et al.* 2014).

In order to carry out the shear stress analysis, the coordinates of center of mass (G) and rigidity (C) of the critical story (i.e. ground story) are calculated as (x_G=5.9 m, y_G=6.5 m) and (x_C=7.0 m, y_C=5.6 m). There is a significant difference between the locations of these two centers, indicating that the shear forces due to torsion can be effective on the overall shear force demand of the structural walls.

Assuming that the base shear acts from the center of mass, inducing a torsional moment about the center of rigidity, the shear forces in a typical wall i due to direct shear ($V_{xi,1}$) plus shear caused by torsion ($V_{xi,2}$) in both orthogonal directions can be calculated by using the following formulations (see Figure 4).

$$V_{xi} = V_{xi,1} + V_{xi,2} = \frac{k_{xi}}{\sum_i k_{xi}} V_{bx} + \frac{M_T}{J} k_{xi} y_i \tag{3.a}$$

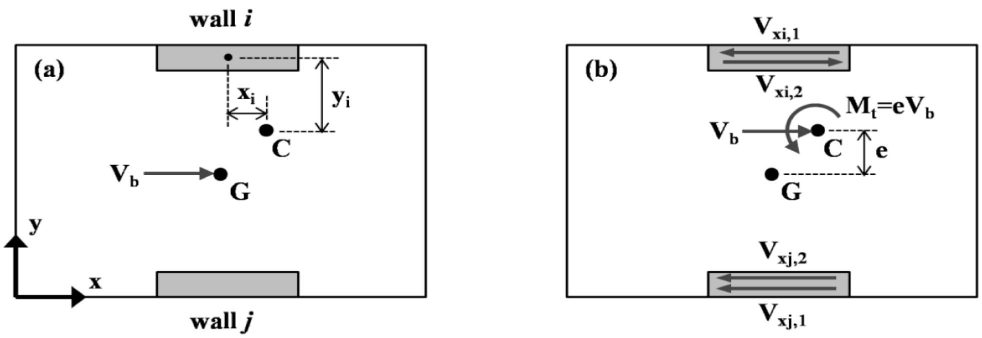
$$V_{yi} = V_{yi,1} + V_{yi,2} = \frac{k_{yi}}{\sum_i k_{yi}} V_{by} + \frac{M_T}{J} k_{yi} x_i \tag{3.b}$$

In these equations, V_{xi} and V_{yi} are the total shear forces in x and y directions, k_{xi} and k_{yi} are the relative stiffnesses of wall i in x and y directions, x_i and y_i are the distances from the center of the wall to the center of rigidity, M_T is the torsional moment and J is the torsional constant defined as

$$J = \sum (k_{xi} y_i^2 + k_{yi} x_i^2) \tag{4}$$

In the last stage, the shear stresses (τ_i) are obtained by dividing the total shear forces to the cross-sectional areas of the walls for the most unfavorable loading condition and then compared with the allowable shear stresses (τ_{em}) as defined by the code.

Figure 4. Definitions used in the shear stress analysis



$$\tau_{em} = \tau_o + \mu\sigma \tag{5}$$

In Equation 5, parameters τ_o and μ stand for allowable cracking stress (obtained as a function of masonry unit and mortar type) and friction coefficient (recommended value by the code is 0.5), respectively. In the case study building, the calculated shear stresses range between 0.13-0.21 MPa in N-S direction and between 0.27-0.37 MPa in E-W direction. When these values are compared with the allowable shear stress that was obtained as 0.20 MPa, it is observed that the structural walls in the E-W direction would be highly overstressed if the building were subjected to design level earthquake (with a return period of 475 years), which is in accordance with the previous observations. If the same analysis is repeated by using a service-life level earthquake (with a return period of 72 years), the maximum shear stresses in N-S and E-W directions are calculated as 0.10 and 0.15 MPa, respectively. Hence, it can be concluded that if the building were subjected to a moderate level earthquake that is probable to occur in the service-life, it would experience some damage due to shear stresses without endangering the overall stability of the building. However in the case that the building experienced the design level earthquake with a less probability of occurrence within the service-life, the building would probably experience severe damage with a high collapse risk.

The calculations performed above assume that structural walls do not have any stability problems. Observations indicate otherwise, there seems to be some major deficiencies. First, the inferior conditions of wall-to-wall and wall-to-floor connections endanger the integrity of the walls and the box-like behavior of the complete building, which is in turn affecting the overall safety of the building. Second, due to their relative flexibility, the inner walls with bağdadi covering do not provide a serious contribution to the lateral load capacity of the building. Hence the building has to rely on peripheral masonry walls, which are not adequate in length to ensure the load carrying capacity of the building. The analysis results indicate that the overall safety of the building is under high risk if a strong earthquake hits the region. Third, the uneven distribution of openings in the peripheral walls induces plan irregularity in the building. Such an irregularity causes additional shear stresses in the walls. Furthermore, the timber floor slabs do not have sufficient rigidity in their own planes. This may cause uneven distribution of lateral forces to the walls, which is very difficult to quantify. Finally, the timber frame elements within the structural walls exhibit significant deterioration due to many reasons as time or weathering effects and insect and fungal activity. These members should be replaced with new members in order to enable the structural walls to regain their load-carrying capacities.

To conclude, the field observations and seismic performance assessment studies in accordance with the current seismic code reveal that the Arditi residence requires restoration and seismic retrofit in order to ensure its structural safety against vertical and horizontal loads. The following section explains the details of the proposed intervention strategy for the building under consideration.

THE INTERVENTION STRATEGY

Based on the observations and analysis of the existing structure, it is decided that both member and structure level interventions are necessary to mitigate the structural risk of the case study building under seismic loading. Purpose of the member level interventions is to bring the members to a condition that the members will be sufficient for the intended structural service. Deterioration of the material, cracking and loosing plumb line are the main causes of the performance lost at the member level. Individual walls, embedded timber framing in the walls and the floor beams are typical members considered. Protective measures and repair or rebuild are possible levels or stages of the member level interventions. Purpose of the structure level intervention is to modify the response of the structure for seismic risk mitigation. Modifying the member connections, the stiffness of the diaphragms, the continuity of the members and the stiffness of the system are some possible structure level interventions.

Member Level Interventions

Observations in the case study building reveal that majority of the timber framing within the stone walls and the floor diaphragms had lost their structural characteristics due to the insect and fungal activity. Therefore, it is decided to replace the structurally unfit members. Pine is the original timber type for these members. Cypress had been used for some of the main girders as well. Members are planned to be replaced using the original timber types. Wherever it is possible, structural capacities are checked according to current design criteria under gravity loads and it is verified that existing cross section satisfy the current design demands. It is stressed in design drawings that the replacement process should be in stages and should have the minimal impact on the original system. Stages of such an approach are provided in design drawings.

As mentioned before, timber tie beams just above the ground level within the basement masonry walls are observed to be totally disintegrated and empty spaces exist in their place (Fig 3a). With the permission of the responsible restorer, these tie beams were decided to be replaced with concrete beams due to the location, type of the wall and the maintenance issues. This is the only location in the building that concrete is used for restoration purpose. Tie beams are spanning all through the basement walls and it is not possible to unload the walls for any purpose. Therefore, it is not an option to cast a single beam spanning all the length. A scheme that permits the casting of the ground beam in stages, considering length and cross section, is selected. Thickness of the wall at this level, which is approximately 60 cm, enables to apply such an approach.

There are many locations in the building with visible cracking on plaster. Cracks with less than 1 mm width are decided to be disregarded and only cosmetic applications are proposed. Intervention decisions for other locations are proposed to be based on controlling the penetration of the observed cracks to the body of stones in the walls. If such penetrations are observed, walls in the region are planned to be removed and rebuild locally. If the cracks are wider than 1 mm and they are following the mortar lines, it is decided to use hydraulic lime mortar injection for the cracks. Extensive cracking and disintegration at some portions of the existing masonry walls necessitates the removal and re-construction. As it can be observed from Fig. 3c and d, north-west corner of the case study building needs such an intervention due to the deterioration, excessive cracking and being out of plumb line.

Structure Level Interventions

Based on the qualitative and quantitative assessment of the case study building, following structure level vulnerabilities are defined:

1. The reinforced concrete frame at the south-east of the case study building, which is constructed at a later date, cause torsional irregularity,
2. Wall to slab connections are not strong enough to ensure satisfactory performance during seismic loading,
3. Floor diaphragms do not have continuity at interior walls crossings,
4. Floor diaphragms are not stiff enough to provide necessary force transfers,
5. Structure is considerably weak along E-W direction under lateral loads.

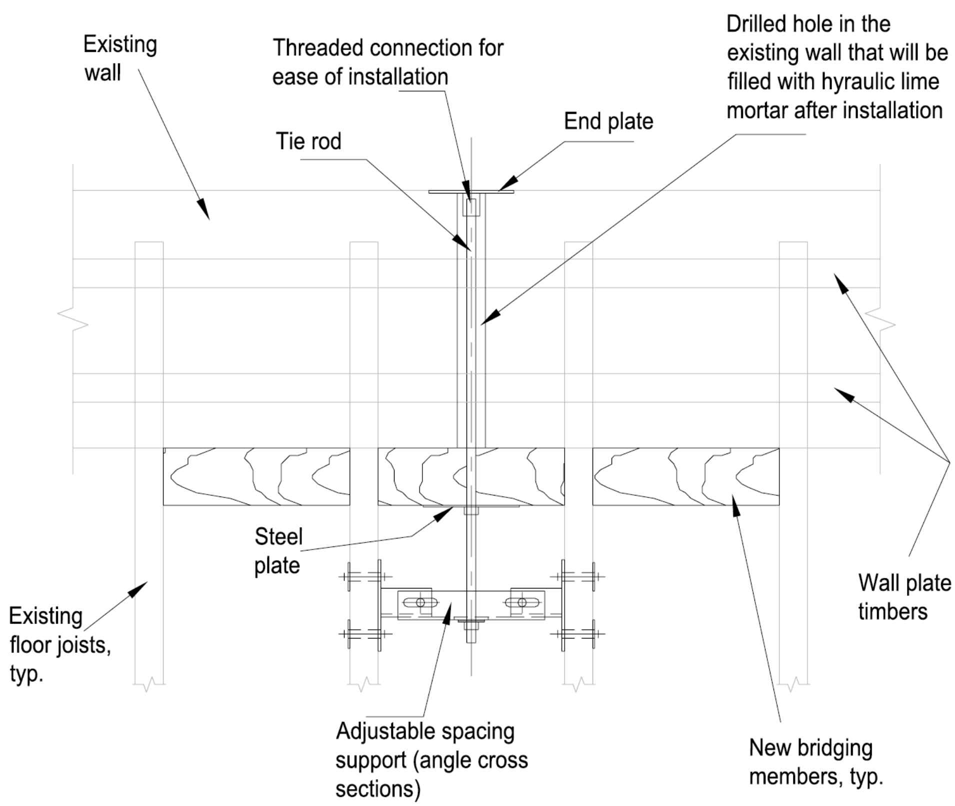
The partial reinforced concrete frame at the south-east of the building, which is added to the original framing at a later date, is inconsistent to the original framing from material and stiffness perspectives. Moreover, its location at the corner of the building is the cause of a serious torsional irregularity in the case study building, Fig. 2. Therefore, it should be removed in order to enhance the structural safety. The space left after removal should be reconstructed in accordance with the original structural layout.

In a masonry structure, sufficient connections among the walls and the floor framing are necessary for controlling the buckling load and the stability of the walls under lateral loads. Structural system of masonry buildings function through bracing of the out of plane walls by in plane walls. Floor framing should provide the necessary force flow paths for this purpose. Naturally, demands from the floor to wall connections reach to higher values under seismic conditions. Existing floor to wall connections are developed by the penetration of the floor joists to the walls. Such connections exist when the joists are perpendicular to the walls. Hence, walls that are parallel to floor joists have such connections only along bridging members. Joists penetrating to the walls are positioned on wall plate timbers that are extending within the walls. Typically, two nails are observed to be applied between the joist and wall plates.

Analysis of the structure with the demands from existing Turkish Earthquake Code indicates that floor to wall connections will be subjected to tension and shear forces together at the worse condition. Noting the difficulty in defining the capacity of the existing connection detail, an approach based on assuming a constant friction between penetrating joists and the wall is selected. This approach provides an opportunity to decide about the most critical floor-wall connection in the building. Tension demand from the connection is calculated by using the acceleration demand defined by Turkish Seismic Code and the tributary areas of the walls. Turkish Seismic Code accepts a seismic load reduction factor of two for masonry structures. Considering that the structure is in the most active earthquake region in Turkey according code, a spectral acceleration of 1.0g (gravitational acceleration) is obtained. Based on the seismic load reduction factor of two, an effective acceleration of 0.5g is obtained. Therefore, along the walls that are working out of plane for the given earthquake direction, inertial loads developed by the calculated acceleration value should be transferred by the slab wall connection. Active mass is accepted to be defined by the tributary areas of the each slab line thru dividing the wall area along the mid height of the each story. By the constant friction coefficient assumption and the loads per length of the floor connections, ground floor connections are singled out as the most critical ones. Shear demand from the connection is calculated based on the torsional moment that is going to be developed due to the eccentricity between the centers of mass and rigidity of the building. Interior walls are typically timber framing only and are relatively flexible compared to the exterior walls. Their contribution to the torsional moment is neglected. Therefore, the torsional resistance of the system is assumed to be developed by the exterior walls only. The results show that even with a generous coefficient of friction (taken to be 0.4) between the joists and the wall, structural system is vulnerable. Therefore, it is decide to rehabilitate the floor frame to wall connections.

The rehabilitation of wall-to-floor connections is provided by using through bolt anchors and anchor plates. The wall-to-floor connections in the ground and first floor are braced by this approach. Such a solution is a variation of tie systems in historical masonry walls (NIKER, 2010). It is also defined as one of the alternative methods for rehabilitation of the existing unreinforced masonry bearing wall buildings by FEMA 547 (2006). The details of the connection are presented in Fig. 5. In order to develop bearing capacity and to protect the rod against the elements, the drilled hole in the wall is designed to be filled with hydraulic lime mortar after the installation. Necessary access holes are provided on the steel plates for this purpose. Since exterior walls have a lime plaster as a final coat, anchor plates in the exterior of

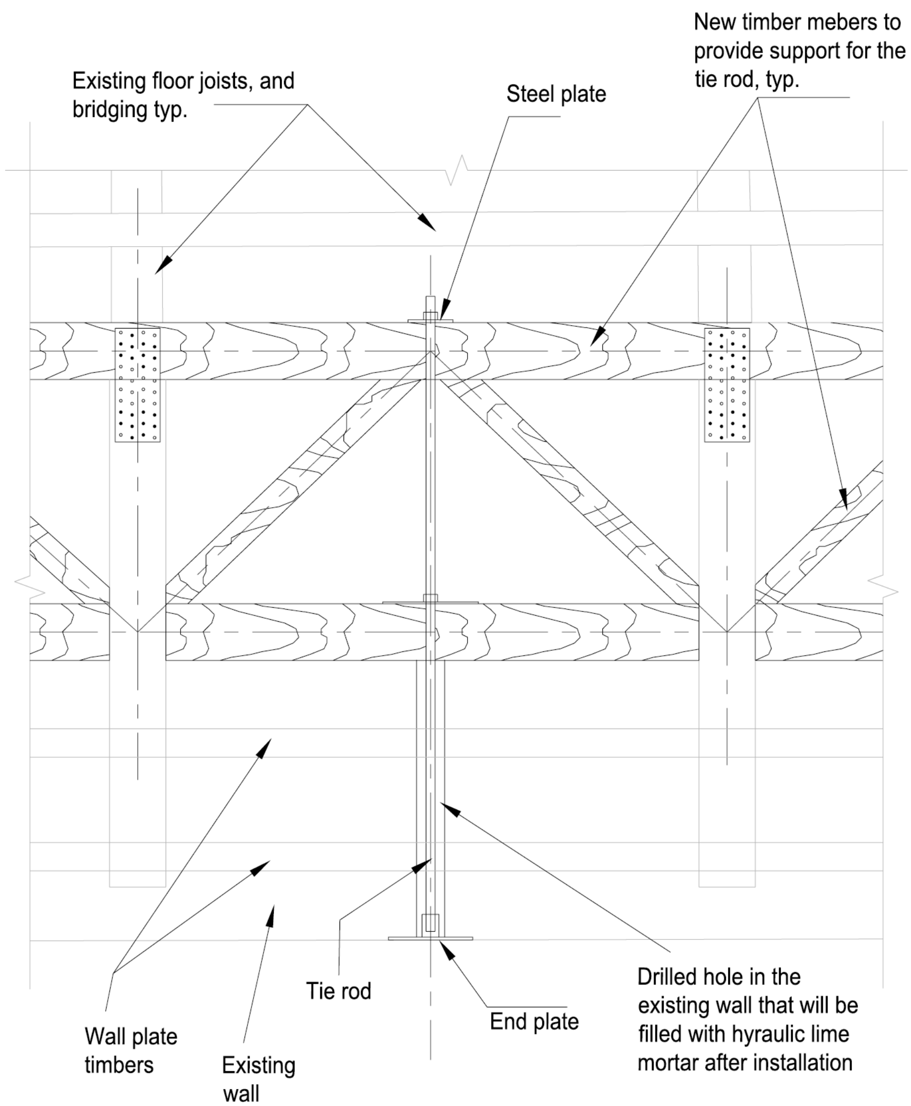
Figure 5. Details of the tie-rods that are connecting the exterior wall to floor framing



Note: Existing wall and the timber is not hatched to have a clear view.

the walls will be covered with plaster after the construction. Considering the existing geometry and the positions of the floor joists, ties with 1 meter intervals are proposed. Based on the shear and tension force calculations, ties are designed to resist a tension force of 6 kN and a shear force of 10 kN. Indicated capacities are sufficient to transfer the seismic loads with the assumption that friction coefficient between the penetrating joists and the wall is 0.4. The exposed steel parts of the planned tie system fit into the spacing between the joists. The sheeting underneath the floor framing will hide it in the end. It should be noted that the provided tie system can be totally uninstalled with minimum damage to original building. Tie detail is adopted to be applied at the exterior walls that are extending parallel to the floor framing. At these edges, timber compression struts are developed within the floor framing to transfer the developing forces (Fig. 6). Another application is carried out for the local areas with no access to

Figure 6. Details of the tie-rods that are connecting to the exterior walls extending parallel to floor framing



Note: Existing wall and the timber is not hatched to have a clear view.

the exterior face of the wall due to the existing build up at the entrance of the building. These rods are designed to be embedded to the holes which will be filled by hydraulic lime repair mortar.

Roof diaphragm requires a different approach at the wall connection since the walls halt at this level. Dovels embedded to drilled holes with hydraulic repair mortar and a peripheral timber plate connecting these dowels is designed to provide the necessary connection on the wall for the diaphragm (Figure 7).

Assessment of the floor framing at the internal wall and girder crossings show that most of these connections are not sufficient to provide necessary force transfer from one side to other. Therefore, noting that at the internal wall crossings floor covering is discontinuous, floor is susceptible to tearing or pull out at these locations (Fig. 8). Change in the orientation of the floor joists magnifies the vulnerability. To mitigate this problem, details that enable force transfer are provided. If the joists are along the same direction on each side of the gap, bolts connecting the joist on each side are designed to provide continuity. If there is not enough room for such a connection, new short timber members, which have the same size with the joists, are used. If the joists are changing direction on each side of the gap, joists or crossing timber are extended enough to provide connection to the first orthogonal joist and the floor covering.

Eccentricity between the centers of mass and rigidity of the building and resulting torsion forces cause a significant demand during transfer of forces from flexible to stiffer regions of the building. Existing floor framing is composed of typical 5x13 cm joists with 30-35 cm spacing. There exists same size bridging with 105 cm typical spacing in the perpendicular direction. Joists are covered with single layer, 2.5 cm thick pine planks in a direction perpendicular to main direction of the joists. Main hall in the ground floor have an additional 5-10 cm layer of mortar and tiles as covering. Existing floor framing is not stiff and strong enough to satisfy the required force transfers. Therefore, a double floor cover with a corresponding nailing scheme is developed. New scheme consists of a 3 cm plywood level and a 2.5 cm pine planks level. Bottom layer is designed to maximize the use of the full size plates. Upper layer is

Figure 7. Details of the tie-rods that are providing connection base for the roof diaphragm

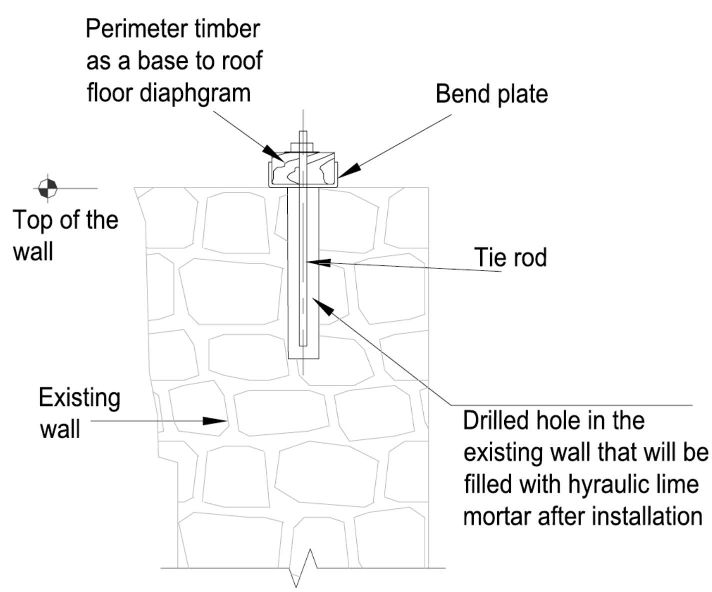
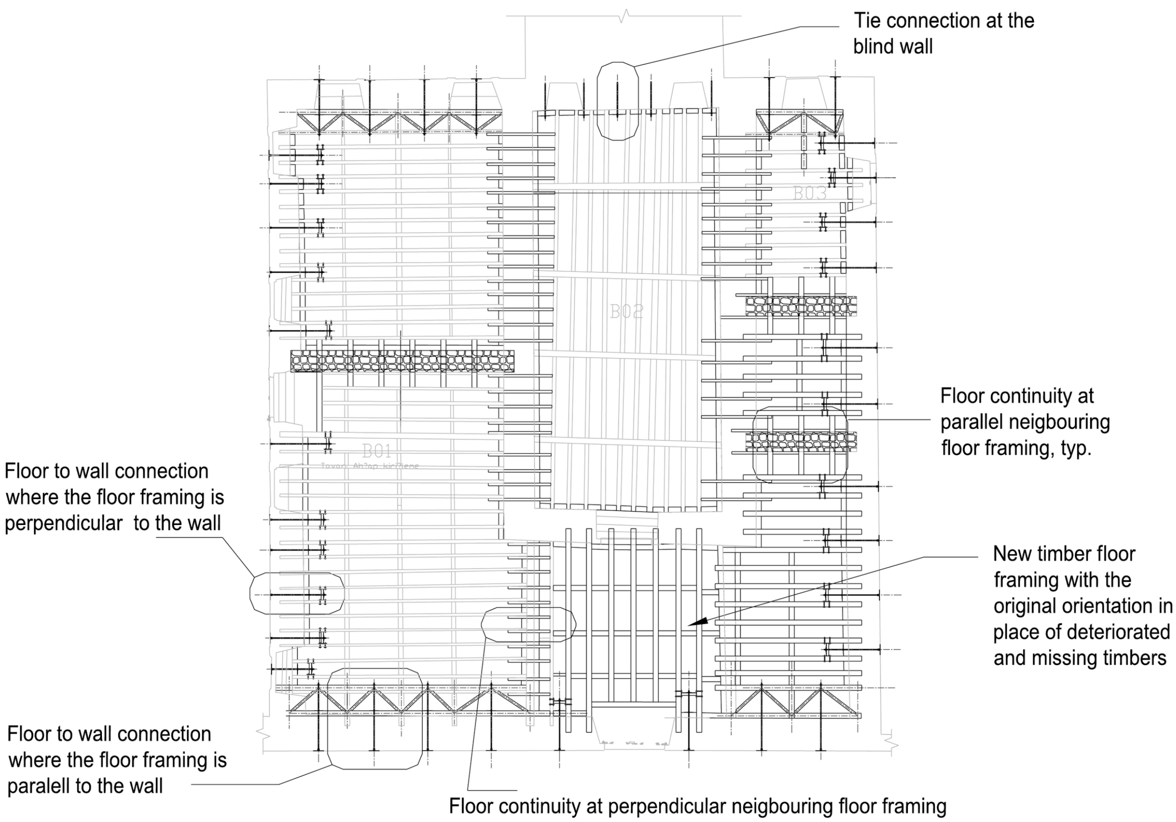


Figure 8. Existing framing of the ground floor diaphragm



designed to have the same orientation with the original floor covering. Therefore, original layout could be preserved and if possible original wooden planks could be used. The new floor framing is calculated to be sufficient in strength and stiffness for the required force transfers.

As presented in field investigation section, existing structure is considerably weak along the E-W direction under earthquake loads. In order to provide an additional capacity, it is decided to strengthen some of the existing interior walls. To achieve this task, similar materials and construction techniques should be used, which is important from conservation point of view. Same material and construction techniques also provide stiffness and strength compatibility between the existing and new construction. Therefore, old and new systems could resist the loads in harmony. Accordingly, some of the internal walls at the selected locations are converted to hımış walls or the timber frame walls are strengthened. The locations of the converted walls are presented in Figure 2. Internal walls at the indicated locations are designed to be rubble masonry walls at the ground floor and hımış walls at the ground floor. Short walls at the east side do not continue at the first floor, only the long wall on the west side continues in the first floor as a strengthened timber frame wall. The original timber framed walls have 18-20 cm thickness. The new hımış walls are designed to be 40 cm thick. Strengthened timber wall is obtained by constructing a parallel frame to original wall frame. Total thickness of the final frame becomes 40 cm.

In order to protect the wall paintings at the north surface of the ground floor long wall during modification, a scheme is selected to permit the construction of the hımış wall from the south face of the wall. The existing timber framing is designed to be filled by rubble stone from south face. Therefore, any complication extending to the north face is minimized.

Analysis of the structure to identify the effect of the modified walls show that maximum shear stresses developed under seismic loads are dropped from 0.37 to 0.22 MPa for E-W direction and 0.21 to 0.20 MPa in the N-S direction. Noting that allowable stress level is 0.20 MPa, well balanced stress levels that is close to allowable level in both directions is satisfied. It should be emphasized that rather than the absolute values of the stresses, which is based on a series of assumptions, the proximity of the values in both directions is important. Therefore, it is essential to avoid a premature failure in one direction before the other direction is approaching its limits.

As a result of the structural interventions, the case study building is expected to gain necessary connectivity for box-like action, have necessary capacity to transfer forces where the resistance exists and have sufficient member sizes to have balanced stress levels in both directions of the structure. Even though the case study building may not have been raised to the performance level that completely satisfies the modern seismic design requirements, it is expected to have an improved behavior under seismic demands.

CONCLUSION

Restoration and sustainability of cultural heritage is a challenging task which becomes more demanding in seismically active regions. The main difficulty in seismic retrofit of a historic building is to develop intervention strategies that provide the required structural safety by having a minimal effect on the historic information carried by the structure. The main objective of the seismic retrofit is defined by Kelly et al. (2011) in a single sentence as "The goal of seismic retrofit is to reduce the potential for heavy structural damage or collapse and not to earthquake proof." Therefore, in this context the purpose appears to be bracing the building to the maximum and balanced capacities that are acceptable from restoration point of view.

The building is located in Urla, at the west coast of Turkey. It had been constructed for one of the prominent families of the region in that era. It is named after the family whom constructed and used the building originally, the Arditi Residence. It is a two story building with a partial basement. It was used as a residence till 1930s, after which converted to a public building. From the structural point of view, the building can be regarded as a mixed system since it is composed of three different techniques. The basement walls had been constructed by rubble stone whereas the periphery walls in the ground and the first floor had been constructed as timber framed stone masonry walls, locally called "hımış". Some of the interior walls in the ground floor and all of the interior walls in the first floor are timber frame with "bağdadi" covering, a local and traditional construction technique similar to timber lath and plaster. In both stories, the structural walls had been connected to a timber floor system.

Assessment and analysis of the building revealed that there are vulnerabilities both at the member and the structure levels. Member level vulnerabilities are due the member deteriorations. Depending on the level of deterioration, the selected intervention strategy is either to repair or to replace the corresponding member. Typically, repairs and replacements take place by using similar types of material with similar size. One exception to this approach takes place for the timber tie beams that are located in the ground level of the basement walls. Due to the existing condition and the long term sustainability, reinforced concrete tie beams are used instead of original timber beams.

Structure level vulnerabilities are due to the obstructed stress paths and impaired structural behavior. The mitigation of structure level vulnerabilities necessitates extensive multi member interventions. The first intervention is the removal of the partial reinforced concrete frame, which is at the south east of the building. This frame is a later addition and it has a detrimental effect by increasing the torsional forces in the building under seismic forces. The next group of interventions is towards ensuring a box like action of the building and providing sufficient flow paths for transfer of developing forces to stronger and stiffer regions of the structure. For this purpose, floor to wall connections are strengthened, floor frame members interconnectivity is provided and rigidity of the floor diaphragms are carried to acceptable levels. The last intervention is increasing the east west resistance of the building to lateral forces. Analysis performed indicated that building is considerably weak in this direction compared the N-S direction. For this purpose stone masonry walls in the basement and ground stories and a timber frame wall at the first floor are added to the building. Except two of the basement walls, new walls are enlarged and/or modified versions of the existing walls.

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KEY TERMS AND DEFINITIONS

Bağdadi: A traditional construction technique in Turkey similar to timber lath and plaster technique.

Hımış: A traditional construction type in Turkey with timber framed masonry walls.

Masonry Building: A type of construction, in which small units are laid over each other to form the primary load carrying system, i.e. structural walls. The walls are connected to each other through horizontal members that act as diaphragms and transfer all different types of loads to the walls.

Member Level Intervention: The retrofit approach to bring the members to a condition that they will have adequate capacity for the intended structural service.

Preliminary Evaluation: An initial phase of investigation for a building, generally in the form of a walk-down survey, to assess the current status of the building in a global sense.

Seismic Performance Assessment: A later phase of investigation for a building, in which a detailed analysis is carried out in order to evaluate the seismic performance of the building.

Structure Level Intervention: The retrofit approach to enhance the performance of the structural system as a whole for seismic risk mitigation.